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FRICTION COURSES FOR MODERATE TRAFFIC HIGHWAYS

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FRICTION COURSES FOR MODERATE TRAFFIC HIGHWAYS

by

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A B S T R A C T

This paper reports on the final evaluation of the performance of 17 bituminous test sections constructed in 1978 on Highway 7 near Lindsay, Ontario.

The objective of the trial was to develop suitable surface friction course mixes for highways carrying moderate volume of traffic (about 5000 AADT) at posted speed limit of 80 km/h. These mixes would provide and maintain adequate levels of surface friction to reduce wet pavement skidding accidents. Both open- and dense-graded type of mixes were included in the evaluation. Two standard mixes were incorporated for control purposes. A specialty mix (a patented design mix called DELUGRIP) was also placed in the trial. Aggregates used consisted of crushed gravels, local sand and screenings of various blends.

Frictional properties of the test sections were measured three times within the first 6-year period. Samples of the surface course mixes were periodically taken for laboratory testing. Also a performance rating system was developed to evaluate the test sections. The system is based on the average friction number values, mix characteristics and field observations.

Friction results indicate that the coarse aggregate content and quality is a major factor for determining the level of friction achievable in a mix. The mixes which were found suitable for moderate traffic are those containing at least 25% of hard igneous coarse aggregate with the coarse aggregate content in the mix greater than 60%. The use of natural sands should be avoided in such mixes.

In general, open friction course mixes using granite/basalt coarse aggregate (without limestone coarse aggregate) were found to perform best, but some of the dense friction course mixes also performed satisfactorily. Mixes containing a high proportion of limestone coarse aggregate from local supplies were found unsatisfactory both in terms of friction number and, in most cases, durability. More specific mix design recommendations were developed for selection of friction course mixes for moderate traffic under southern Ontario climatic conditions.

RÉSUMÉ

Cet article présente l'évaluation finale du rendement de 17 sections d'essai en bitume construites en 1978 sur la route 7 à proximité de Lindsay, en Ontario.

L'objectif de cet essai était de mettre au point des mélanges convenables pour les chaussées supportant une circulation modérée (de 5,000 à 10,000 AADT [circulation quotidienne moyenne annuelle]) à des vitesses limitées à 80 km/h. Ces mélanges doivent procurer une surface de friction suffisante pour réduire les accidents par dérapage sur chaussée mouillée. L'évaluation a porté sur deux types de mélanges différents, deux mélanges standard y étant ajoutés à titre de référence. Un mélange spécialisé (mélange breveté appelé DELUGRIP) a aussi été soumis à cet essai. Les agrégats utilisés consistaient en gravier concassé, sable d'origine locale et menues particules de différents mélanges.

Les propriétés de friction des sections d'essai ont fait l'objet de trois mesures au cours de la première période de 6 ans. Des échantillons de revêtement ont été prélevés à différents moments pour essais en laboratoire. Un système d'évaluation du rendement a aussi été mis au point pour permettre l'évaluation des sections d'essai. Ce système est basé sur les valeurs moyennes de friction, les caractéristiques du mélange et les observations faites sur place.

Les résultats de friction indiquent que la teneur en agrégats grossiers et la qualité de ces agrégats sont des facteurs fondamentaux du niveau de friction d'un mélange. Les mélanges convenant à une circulation modérée sont ceux qui contiennent au moins 25% d'agrégats ignés grossiers, la teneur en agrégats grossiers du mélange dépassant 60%. Dans de tels mélanges, il convient d'éviter l'utilisation de sables naturels.

En général, les mélanges de friction du type ouvert à base d'agrégats grossiers granitiques/basaltiques (sans agrégats grossiers calcaires) se comportent mieux, même si certains des mélanges denses ont aussi donné un rendement satisfaisant. Les mélanges contenant un pourcentage élevé d'agrégats grossiers calcaires provenant de carrières locales n'ont pas donné satisfaction, tant en termes de friction que, dans la plupart des cas, de durabilité. Des recommandations plus précises ont été préparées pour la sélection des mélanges de friction soumis à une circulation modérée dans les conditions climatiques du sud de l'Ontario.

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1. INTRODUCTION

The Ministry initiated a research and development project in 1977 to develop hot laid surface course mixes with high frictional qualities. The main objective of the project was to determine more economical friction course mixes for locations other than heavily trafficked high-speed freeways without resorting to the use of scarce, premium quality aggregates.

In particular, answers to the following questions were sought for:

Can we improve on the friction properties of mixes using marginal aggregates available locally?

What level of improvement can we expect of blending better quality aggregates and at what extra cost?

Would the open-graded surface course mixes using local limestone aggregates provide adequate frictional resistance and durability?

With the above points in mind, 17 bituminous surface course mixes were designed and constructed in September, 1978. The test site was monitored for seven years. This report summarizes the work done on field observation and laboratory evaluation of the performance of the test mixes over 7 years of service.

2. MATERIALS

2.1. Aggregates

The aggregates employed for the trial are commonly available materials in the Province of Ontario. Coarse aggregates were of igneous gravels from the north and limestone from the south of the Province. Fine aggregates were of local sand, local limestone screenings, and igneous screenings from northern Ontario.

The aggregates used in the test mixes were as follows:

COARSE AGGREGATES

- * GRANITE/BASALT GRAVEL
from Maple Ridge
- * LIMESTONE GRAVEL from
Beamish-Brown
- * TRAPROCK STONE from
Havelock

FINE AGGREGATES

- * SCREENINGS, washed/unwashed from
Maple Ridge and Beamish-Brown
- * NATURAL SAND from Ormell
- * LIMESTONE FILLER

A brief description of these materials and some properties of the coarse aggregate are given in Table 1. The Maple Ridge igneous material is similar in characteristics to Havelock traprock.

The fine aggregate was from the same source as the coarse aggregates. Washed and unwashed fine aggregates and local natural sand were utilized.

2.2. Asphalt Cement

An 85/100 penetration grade asphalt cement was used for all the test mixes except mix No.16 for which 60/70 penetration grade was obtained from Gulf Clarkson refinery. The penetration value of the original asphalt was 90 for the 85/100 grade, and 54 for the 60/70 penetration grade.

2.3. Filler

Filler was used in mix No.16 only. It was of the limestone type with a gradation conforming to the MTO specification (min. 80% passing .075mm sieve).

3. MIX DESIGNS

There were several factors considered during the selection of the experimental mixes. Among them, previous MTO experiences with friction course mixes placed in the test sections on Highway 401 Toronto By-Pass [1] were taken into account. As well, technology on friction course mixes from other jurisdictions were also considered.

The selected test mixes included:

- a) Six open friction course (OFC) mixes - 65% of Coarse Aggregate (C.A.) and washed screenings as Fine Aggregate (F.A.).
- b) Eight dense friction course (DFC) mixes - 55% of C.A. and various blends of F.A..
- c) Two standard mixes - HL-3 and HL-1 containing 45% of C.A. and 55% of local sand.
- d) DELUGRIP mix - designed by Dunlop Ltd.

- * Placement of the test mixes was carried out in good weather conditions (mid-September) and was completed in 5 days.
- * Open mixes (1 to 6) and mix No.16 were compacted using a 10-ton steel-wheeled roller only. Both steel and rubber-tired rollers were used on all other test sections. The paver was equipped with a vibratory screed.
- * No special problems were encountered in placing the mixes.

6. POST-CONSTRUCTION MEASUREMENTS

Water permeability and ASTM brake force trailer measurements were carried out to determine initial water drainage capability and frictional properties of the experimental mixes respectively.

Permeability test was carried out within the first week after construction, using procedure developed by the Johns-Manville Co. [4].

Results showed that all of the mixes were too permeable to measure because of the rapid water drainage (>25 ml/min is considered permeable) with the exception of:

- Nos. 15 and 17 (control mixes), which were impermeable, and
- Nos. 7 and 8, which gave a result higher than 275 ml/min.

Surface friction measurements were carried out for the first time one month after construction, in October, using a skid trailer conforming to ASTM Standard E274, at 50 and 80 km/h. Friction Number (FN) was determined for each test section at the two speeds. The results are given in Table 3.

7. FIELD SAMPLING AND OBSERVATIONS

7.1. Sampling

Mix samples were taken both at the plant (at discharge) and from the job site after placement of the mix but before compaction. The samples were tested in the laboratory for mix compositions and Marshall values on recompacted mixes.

Pavement cores were taken from each of 17 test sections within a week after construction of the test sections as well as at the 3rd, and 7th year of service. These samples were tested for:

- mix gradation and asphalt cement content,
- penetration and viscosity of recovered A.C.,
- Marshall properties of recompact mix,
- pavement compaction.

7.2. Observations

Detailed survey of the test sections was carried out after 6 years of service to establish if there are any relationships between laboratory test results and field observations. Table 4 gives a summary of the crack map data gathered during the field survey. It includes longitudinal, transverse and other types of cracks. The pavement surface "Crack Index", based on crack severity and crack type weight factors, was introduced for comparing the performance of different sections. The index was derived from the Distress Manifestation Index [5] and relates to crack damages only. Total Crack Index was calculated and ranking numbers were assigned to each of the sections. Also, a quotient of overall crack length to the length of each test section is shown in the table.

It can be seen that the OFC sections showed much less transverse cracking than most of the DFC sections. The difference could be due to variations in the mixes and better base supports on which these mixes were laid. It can be expected that, apart from the mix properties, the poor pavement base stabilities can account for the increased incidence of crackings and roughnesses.

The test section with a good ranking is No. 5 (i.e. ranked 1 in Table 4) which is followed by No. 2. The poorest ranked mixes in respect to cracking are placed in sections No. 16 (DELUGRIP) and 10 (DFC with 100% limestone C.A.). The heavy sand ravelling and cracking found during the survey in those two sections had resulted in the harsh surface texture leading to an increase in the Friction Number (FN) values.

8. LABORATORY EVALUATION

8.1. Aggregates

As the type of aggregates used in a mix determines the frictional properties and durability of a pavement surface, factors such as aggregate abrasion, susceptibility to polishing, absorption, gradation, nominal size, percentage crushed, particle shape and cleanliness need to be carefully considered during mix designs. Some of these factors and their relationships to friction and durability are examined in the following evaluation.

The performance of aggregates was evaluated by the change in their gradations over the years of service. It was found that mixes containing relatively soft coarse aggregate (e.g. limestone) had the most change in gradation (mixes No. 2,5,8,10), whereas mixes containing very hard coarse aggregates changed very little (mixes No. 3,6,7,9,15,17).

The changes in gradation could occur both internally in the matrix due to degradation and cracking and externally by the action of tire wear and weathering of the aggregates. The effects of these factors were observable in the field where severe ravelling took place in sections No. 2,5,8, and 10. These mixes had the highest changes in gradation curves (Figs. 2,3,4).

8.2. Asphalt Cement

Asphalt cement content of test mixes from samples taken during the monitoring period was within the $\pm 0.3\%$ deviation limits. Due to aggregate absorption and weathering effects, the amount of A.C. extracted from the mixes was slightly lower than the initial results, i.e. average change was:

- OFC mixes: - 0.6%,
- DFC mixes: - 0.3%,
- DELUGRIP: - 1.0%,
- Control mixes: - negligible.

The hardening effects of the A.C. after 7 years are reflected in the retained penetration and the increased viscosity. These changes are shown below:

MIX TYPE	PENETRATION (% ret.pen.)	VISCOSITY (% increase)	HIGHEST CHANGE
OFC	30.0	168	Mix No. 5
DFC	46.5	91	Mix No. 10
DELUGRIP	35.7	362	-----
CONTROL	75.2	18	Mix No. 15

As asphalt cement aging is closely related to the air void content of asphalt mixtures, the biggest change took place in mixes with high void content and high proportion of limestone aggregates (Figs. 5 and 6). DELUGRIP mix, containing a harder original A.C. had a percentage of retained penetration between the OFC and DFC. For mixes with the same air void content, variations in penetration and viscosity values obtained could be due to the hardening of A.C. by temperature fluctuations during the production of the mixes.

The aging in the asphalt cement in mixes No. 5, 6, and 10 was the worst whereas the least occurred in control mixes (HL-1 and HL-3). The A.C. in mix No. 7 aged much less than in all other dense mixes because of the lower air void content.

8.3. Mix Properties

The mix properties changed with time at different degrees during the 7 years of service. Marshall test results on recompacted mixes are summarized in Table 5. In an effort to determine the optimum mixes for durability and friction, factors such as V.M.A., stability, A.C. content, C.A. content, interrelation between Marshall stability, voids in mineral aggregate, and the optimum A.C. content are examined.

The recompacted air void content of the mixes increases relative to the original voids at construction by an average of 3%. The increase is slightly higher for OFC mixes due to more hardening of the A.C. (Figs. 5 and 6). The control mixes (Nos. 15 and 17) were the least susceptible to weathering and they had the lowest increase in voids at the 7th year.

From the results, a steady increase in stability is observed, during the first three years (Table 5). However, some of the mixes (Nos. 2, 3, 5, 7, 8, 15, 17) had lower values at the seventh year because of the changes in gradation and variations in sampling locations. Consequently, some of the Marshall stiffnesses dropped below the initial levels. The DELUGRIP mix became stiffer by approximately 126%. These changes reflect the poorer potential of the mix to resist cracking and

subsequent deterioration.

It is found that the relationship between Marshall design stability and the void content in mineral aggregate (VMA) can be used as an indicator for frictional performance of surface courses. As illustrated in Fig. 7, there is a good correlation between the VMA and mix design stability values (Correl. Coeff. $r=.87$). This is an indication that within the range analyzed, statistically about 75% of the changes in stability can be attributed to the change in % VMA. Fig. 7 shows that mixes with $FN_{80} > 30$ are located above the line drawn for the graph between stability and VMA. The equation is:

$$\text{STABILITY (kN)} = 23 - \text{VMA (\%)}$$

It must be stressed that the relationship is developed based on the results obtained from mixes used in this trial only. This equation or a similar one may also be applicable to other mixes.

9. FRICTIONAL CHARACTERISTICS

9.1. Friction Number and Mix Type

Further to the initial measurement at one month after construction, testing of frictional properties was performed after the second year and sixth year of pavement service. The Breaking Force Trailer was used and the FN values were taken at 80 km/h only (Table 6). The following shows the changes from the initial FN values for the different types of mixes:

MIX TYPE	At 2 yrs.	At 6 yrs.
OFC (#1 to 6)	-1.15	+2.62
DFC (#7 to 14)	-2.26	+0.99
HL-3 (#15)	-4.90	-1.90
HL-1 (#17)	-0.20	+6.40
DELUGRIP (#16)	-4.30	+1.25

OFC mixes had an average FN value of about 1.6 units higher than DFC mixes at the 6th year but lower by 1.1 units in the 2nd year. The results show that friction level of $FN_{80} > 30$ can be obtained from both OFC mixes (Nos. 1 and 6) and DFC mixes (Nos. 11, 12, 17) (Fig. 8).

9.2. Friction Number vs Coarse Aggregate in the Mix

Friction values obtained varied with different mix compositions. For the two periods monitored (2nd and 6th year) the greatest overall decrease in FN, ranging from 1 to 5 units, took place on test sections Nos. 2, 5, 7, 8, 14, and 15, where limestone coarse aggregate and limestone and/or sand fine aggregate were used (except for mix No. 2). The highest relative increase in FN value over these years occurred in test sections No. 4, 6, and 17. All of them contained hard coarse aggregate and relatively soft fines. There was no significant change in frictional values for other test sections.

Mixes containing more of the crushed aggregates performed better in respect to frictional properties. A correlation coefficient of 0.71 (Fig. 5) is obtained between FN and % crushed C.A.. This shows that more than 50% ($r^2 \cdot 100\%$) change in FN_{80} can be directly related to the crushed coarse aggregate content in the mixes. In general terms and within limits the correlation means that, in order to increase the FN_{80} by 1 unit (at 6th year) an increase of crushed coarse aggregate content by about 1.4% is required.

All of the mixes with good quality crushed coarse aggregate at content of >50% had $FN_{80} > 36$ after 6 years of service. Mixes containing limestone coarse aggregate with low content of crushed particles (i.e. <40%) and natural sand had FN_{80} value <30. Fig. 8 illustrates a strong dependence of friction level at both 2nd and 6th year of service on the content of hard crushed gravel in the coarse fraction of the mix. Fig. 8 also shows that, in order to provide friction level of $FN_{80} > 30$ throughout the 6 years period monitored, the percentage of crushed igneous coarse aggregate content in a mix should be at least 25%.

10. CONCLUSIONS

- <1> Imported premium quality aggregates (e.g. Maple Ridge or Havelock) can significantly improve the performance of surface friction courses. For moderate traffic roads, the coarse aggregate should contain at least 25% of hard igneous aggregate in the total aggregate mix.
- <2> Sands and limestone available locally (as those used in the experiment) in southern Ontario are not considered suitable for use alone in asphalt mixes for moderate traffic roads to provide satisfactory frictional characteristics.
- <3> Open-graded mixes employing 100% of local aggregate (limestone, natural sand) did not perform satisfactorily neither in terms of friction nor durability. However, open mixes generally achieved slightly higher friction values than

dense mixes using the same local aggregates.

- <4> OFC mixes containing:
 - about 65% of coarse aggregate in total aggregate mix,
 - at least 25% of high quality coarse aggregate in total aggregate mix, and
 - washed screeningsperformed the best among the 17 mixes.
- <5> Both open and dense friction course mixes can be designed to provide satisfactory level of friction over a long period of time. Mix No. 11 (dense, without limestone) is an example of a good dense friction course.
- <6> Control mix (HL-1), composed of traprock coarse aggregate and natural sand, performed well both in terms of frictional properties and durability.
- <7> DELUGRIP mix, with blends of hard and soft coarse aggregates and screenings as fine aggregate, performed satisfactorily initially but cracked severely at the end of 7-year period. The friction values remained relatively high.

11. RECOMMENDATIONS

Mixes with the best achievable frictional properties could be costly due to the need for importing high quality aggregates. It has been found elsewhere [6] that wet accident rate at AADT in the range of 5,000-10,000, on rural (80 km/h speed) highways, remains relatively insensitive to FN values. In this context, the frictional properties of asphalt mixes should not be considered as a major priority in mix designs (except for accident black spots). Instead, friction property should be considered as equal important as other factors such as durability.

In view of this and based on the results of the experiment, the following general guidelines are recommended for the design of friction mixes for moderate traffic highways:

1. Avoid the use of all soft limestone aggregates in both the coarse and fine aggregates. However, if this is not possible, a mix should contain at least 25% of blended crushed hard-rock coarse aggregate (>4.75 mm) in the total mix in the coarse fraction.
2. Continue the current practice of using a softer asphalt cement grade (e.g. 85/100 in southern and 150/200 in northern Ontario) to prevent the premature cracking and deterioration of a surface course, especially on a weak base.

3. The following relationship can be used as a guide for selections of mixes (using an 85/100 A.C. grade) for frictional properties when other mix design criteria are met:

$$\text{MARSHALL STABILITY (kN)} > 23 - \text{VMA (\%)}$$

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Table 1. CHARACTERISTICS OF COARSE AGGREGATES

Item	PROPERTY	MAPLE RIDGE	BEAMISH	HAVELOCK
1	General Description:	(Gravel)	(Gravel)	(Traprock)
	a) Type of rock/stone	GRANITE/BASALT	LIMESTONE	BASALT
	b) Size used, mm	9.5 or 13.2	9.5 or 13.2	13.2
	c) Source attributes	Very hard, fine-grained crushed gravel	Approved local crushed gravel (relatively soft)	Very hard fine-grai- -ned, cru- -shed rock
	d) Crushed material (% by wt) <LS607>	82	75	100
	e) Petrographic Number(PN) <LS609>	104	114	103
2	Los Angeles Abrasion Value (500rev), %loss <LS603>	16	26	12
3	Magnesium Sulphate Soundness (5 cycles), %loss <LS606>	.7	3.0	.7
4	Water Absorption, % by wt <LS604>	.5	.9	.6
5	Polished Stone Value (PSV) <BS812>	46	43	46
6	Aggregate Abrasion Value (AAV) <BS812>	2.2	Not tested	2.2

LS = MTO Laboratory Standard

BS = British Standard

Table 2. MIX DESIGN GRADATIONS: AS DESIGNED AND AFTER 7 YEARS IN PAVEMENT

Test Section No.	Aggregate Type		Passing Sieve Size (mm), % By Wt										
	COARSE	FINE	.075	.150	.300	.600	1.18	2.36	4.75	6.7	9.5	13.2	
1	M	M	1.4	3.0	6.0	10.0	14.0	22.0	35.0	63.0	98.0	100	
			4.2	7.5	12.6	18.0	23.0	31.0	43.0	63.0	95.7	100	
2	L	M	1.7	4.0	8.0	12.0	17.0	25.0	35.0	59.0	95.0	100	
			4.0	7.0	11.3	16.2	22.0	32.0	49.1	73.0	97.0	100	
3	ML	M	1.9	4.0	8.0	12.0	18.0	25.0	35.0	61.0	97.0	100	
			3.6	6.6	10.8	15.5	21.0	29.8	45.0	67.3	96.2	100	
4	ML	L	1.9	3.0	6.0	10.0	16.0	24.0	35.0	63.0	96.0	100	
			5.0	8.3	12.1	16.2	21.4	30.2	47.9	71.3	98.0	100	
5	L	L	1.8	3.0	6.0	10.0	16.0	24.0	35.0	62.0	95.0	100	
			4.6	7.6	11.3	16.5	22.2	31.6	48.9	73.8	97.0	100	
6	M	L	2.1	4.0	6.0	10.0	14.0	22.0	35.0	64.0	97.0	100	
			4.7	8.1	12.2	16.9	22.1	30.1	44.8	69.0	97.7	100	
7	L	S	1.8	3.0	8.0	17.0	26.0	35.0	45.0	57.0	86.0	100	
			3.0	5.5	11.1	19.2	27.4	36.6	49.2	64.5	85.9	100	
8	L	SL	3.4	6.0	11.0	18.0	26.0	35.0	45.0	56.0	81.0	100	
			4.7	7.8	12.9	19.3	26.7	36.4	52.5	67.7	86.5	99.7	
9	L	SM	3.4	6.0	11.0	18.0	26.0	35.0	45.0	56.0	81.0	100	
			3.7	6.5	11.3	17.6	24.8	34.6	50.7	65.0	88.0	100	
10	L	M	4.6	7.0	12.0	17.0	23.0	33.0	45.0	55.0	80.0	99.0	
			4.7	7.7	11.8	16.7	22.9	33.5	52.5	66.6	88.4	100	
11	M	M	4.0	6.0	10.0	15.0	21.0	30.0	45.0	58.0	78.0	99.0	
			5.1	8.2	12.6	17.3	23.0	32.2	47.4	58.8	78.9	99.2	
12	M	L	5.3	8.0	12.0	17.0	24.0	33.0	45.0	61.0	80.0	99.0	
			6.8	10.2	14.3	18.8	23.8	31.9	47.5	59.3	77.9	98.9	
13	ML	L	5.3	8.0	12.0	17.0	24.0	33.0	45.0	61.0	83.0	100	
			6.4	9.8	14.2	18.9	24.7	33.8	50.9	65.4	84.4	99.8	
14	L	L	5.5	8.0	12.0	18.0	24.0	33.0	45.0	59.0	83.0	100	
			6.3	9.9	13.5	18.1	23.6	32.7	51.7	66.8	87.3	100	
15	L	S	1.2	4.0	11.0	22.0	33.0	44.0	55.0	66.0	85.0	100	
			2.9	6.0	12.8	22.9	33.4	43.7	55.7	66.6	83.1	100	
16	ML	M	6.9	Data not					37.0	available			
			4.5	8.7	12.8	17.4	21.9	28.1	40.0	58.5	85.4	100	
17	T	S	1.2	4.0	12.0	23.0	34.0	45.0	55.0	62.0	82.0	100	
			2.5	5.5	12.3	22.1	32.3	42.9	54.1	62.3	82.6	100	

SYMBOLS:

Coarse Aggregate

M - Maple Ridge Gravel
 L - Beanish (Brown) Limestone
 T - Havelock Traprock

Fine Aggregate

M - Maple Ridge Screenings
 L - Beanish (Brown) Limestone Scr.
 S - Ormell Sand

NOTES:

- 1) Upper values represent design gradation, lower - core sample at 7 year
- 2) 9.5 mm max. size coarse aggregate was used for test sections 1 to 6 (open mixes) and 16 (Delugrip).
- 3) Washed screenings were used for test sections 1 to 6.
- 4) ML, SL, SM are 1:1 blends of respective aggregates, except for section 16 where M to L proportion used was 1.7 to 1.

Table 3. POST-CONSTRUCTION FRICTION NUMBER (FN) VALUES

Test Section No.	FN at 50 km/h			FN at 80 km/h		
	W/B Lane	E/B Lane	Both Lanes	W/B Lane	E/B Lane	Both Lanes
1	47	44	46	37	36	37
2	42	39	41	35	34	35
3	44	43	44	36	35	35
4	40	41	41	33	30	32
5	38	38	38	31	30	31
6	44	44	44	33	35	34
7	43	38	41	30	32	31
8	40	38	39	29	30	29
9	38	39	39	30	31	30
10	44	43	44	33	35	34
11	48	48	48	37	37	37
12	46	45	46	35	35	35
13	44	44	44	33	33	33
14	41	40	41	28	30	29
15	40	38	39	29	27	28
16	48	48	48	38	37	37
17	46	45	46	34	33	33

Table 4. PAVEMENT CONDITION RANKING RESULTS

Test Sec- tion No.	Pavement Surface Crack Index*				Crack per Section Length (m/m)	Surface Condition Ranking**
	P E R C R A C K T Y P E					
	Longitu- -dinal	Trans- -verse	Other Cracks	TOTAL		
1	265.0	8.5	0.0	273.5	1.89	5
2	132.5	74.0	0.0	206.5	1.79	2
3	299.0	70.0	6.0	375.0	2.54	7
4	199.0	7.0	3.0	209.0	0.88	3
5	143.0	0.0	14.5	157.5	0.98	1
6	175.5	0.0	91.0	266.5	1.65	4
7	160.5	50.0	272.0	482.5	2.74	11
8	217.0	27.5	93.5	338.0	1.84	6
9	205.5	190.5	59.5	455.5	1.90	9
10	443.0	191.5	114.0	748.5	3.21	16
11	325.0	84.0	1.5	410.5	1.83	8
12	456.0	110.0	75.5	641.5	3.05	14
13	376.5	158.0	125.5	660.0	2.84	15
14	332.0	144.0	81.0	557.0	2.78	13
15	88.5	282.5	121.0	492.0	2.25	12
16	810.5	341.5	238.0	1390.0	5.30	17
17	51.0	136.5	287.5	475.0	1.42	10

*) The index is defined as:

$$\text{SUM}[\text{SUM}(\text{crack length} * \text{severity weight factor})] * \text{type weight factor}$$

**) The test sections are ranked by number from 1 to 17 based on the crack index; 1 represents the highest rank, 17 - the lowest

Table 5. RECOVERED ASPHALT CEMENT AND ASPHALT MIX CHARACTERISTICS AT DIFFERENT YEARS OF PAVEMENT SERVICE

Test Section No.	Years	RECOVERED A.C.			RECOMPACTED MIX				Voids in Pavement %	Pave-ment Compa-ction %
		Con- tent %	Pene- trat. (25C)	Kinem. Viscos. mm ² /s	Voids in Mix %	MARSHALL Flow Stability mm	Stiffness (Quotient) N/mm			
1	0	5.30	49	496	5.0	3.9	10050	2584	16.5	87.6
	*1	4.92	37	665	5.8	4.0	17347	4294	14.7	90.5
	**3	5.07	21	818	7.8	6.0	16867	2802	14.9	92.3
2	7	5.07	17	1202	6.9	6.0	17004	2848	8.6	98.1
	0	4.70	45	471	5.7	3.6	10890	3059	18.1	86.5
	1	4.41	30	764	6.9	4.0	19260	4803	17.1	89.0
3	3	4.29	22	1093	9.2	3.7	21924	5909	15.0	93.7
	7	4.25	18	1252	7.4	6.4	14345	2259	10.2	97.0
4	0	5.50	46	545	4.6	4.1	10350	2518	16.2	87.7
	1	5.11	40	600	5.4	4.9	18037	3681	13.9	90.9
	3	4.95	31	798	7.6	3.8	19002	4948	15.5	91.7
5	7	4.80	14	1166	7.2	6.4	13195	2078	9.5	97.5
	0	4.60	45	561	5.2	3.5	11960	3437	17.9	86.6
	1	4.22	35	704	5.7	3.2	16849	5298	17.0	88.0
6	3	4.21	24	956	7.0	3.5	21991	6319	14.9	91.7
	7	3.96	17	1362	10.1	5.1	17374	3400	10.8	99.1
	0	4.30	44	565	5.6	3.6	10550	2963	17.9	87.1
	1	3.77	31	741	7.3	3.4	15977	4658	16.5	90.0
	3	3.82	24	1116	8.4	3.5	24339	7055	16.0	91.3
	7	3.64	4	1917	9.8	4.1	15679	3787	10.8	98.9

(Continued)

Table 5 (Continued)

Test Section No.	Years	RECOVERED A.C.			RECOMPACTED MIX			Voids in Pavement %	Pave-ment Compa-ction %
		Con- tent %	Pene- trat. (25C)	Kinem. Viscos. mm ² /s	Voids in Mix %	MARSHALL Flow mm	Stiffness (Quotient) N/mm		
6	0	4.60	40	590	5.4	3.9	10765	18.7	85.9
	1	4.24	35	684	7.2	3.2	13802	17.8	88.6
	3	4.36	27	959	9.2	3.5	18619	14.9	93.8
7	7	4.03	9	1756	9.1	4.7	15790	10.4	98.5
	0	4.80	47	530	2.3	3.9	12554	9.0	93.2
	1	4.65	36	613	2.8	3.5	16128	8.8	93.9
8	3	5.00	38	619	3.0	3.3	18570	7.9	94.8
	7	4.64	31	687	4.0	3.7	14892	6.6	97.3
9	0	4.30	45	556	2.6	4.2	13341	10.6	91.6
	1	4.45	36	623	3.8	3.4	17334	10.2	93.5
	3	4.09	28	881	4.2	4.0	21128	11.5	92.4
10	7	4.32	25	859	4.5	4.0	16436	6.0	97.0
	0	4.50	44	565	4.1	3.7	14247	12.8	90.7
	1	4.46	40	643	3.6	3.6	19460	14.1	91.7
11	3	4.43	25	865	4.6	3.5	21662	11.6	92.6
	7	4.24	23	940	6.3	4.2	19583	9.1	97.1
	0	4.60	43	569	4.3	4.3	13446	14.3	90.0
11	1	4.55	38	667	4.7	3.7	18014	14.5	89.7
	3	4.32	27	894	5.7	3.9	22783	13.0	92.3
	7	4.42	10	1495	8.1	5.6	18895	9.6	98.4
11	0	5.30	48	526	3.2	4.5	12693	13.8	89.0
	1	5.21	33	697	3.7	5.0	19571	12.8	90.5
	3	5.13	28	806	4.4	3.9	17961	11.8	92.3
	7	4.78	18	1081	5.6	6.0	17503	9.2	96.6

(Continued)

Table 5 (Continued)

Test Section No.	Years	RECOVERED A. C.			RECOMPACTED MIX			Voids in Pavement %	Pave-ment Compa-ction %
		Con- tent %	Pene- trat. (25C)	Kinem. Viscos mm ² /s	Voids in Mix %	MARSHALL Flow Stability mm	Stiffness (Quotient) N/mm		
12	0	4.40	46	536	2.8	5.3	13612	14.4	88.1
	1	4.45	38	705	3.9	4.2	18352	12.7	91.0
	3	4.19	25	927	8.8	4.0	16942	14.7	93.6
13	7	4.07	21	1196	5.6	5.2	20127	8.6	96.8
	0	4.30	46	599	3.2	4.8	12843	13.0	89.9
	1	4.37	33	694	4.1	3.9	19238	12.5	91.0
14	3	4.28	33	758	6.6	3.6	17903	11.0	95.3
	7	4.01	18	1154	5.8	5.3	18270	8.2	97.4
15	0	4.30	42	564	2.1	5.1	11877	11.9	90.0
	1	4.52	39	675	2.4	4.5	18966	9.0	93.3
	3	4.31	30	766	4.1	4.1	20794	9.7	94.2
16	7	4.13	50	620	5.9	5.3	18549	7.6	98.3
	0	5.20	50	518	2.2	3.6	12596	7.6	94.5
	1	5.17	47	563	2.5	3.3	20536	6.8	95.5
17	3	5.43	39	599	3.5	3.2	17667	6.5	97.0
	7	5.27	34	614	3.1	3.4	16569	5.0	99.3
18	0	4.70	28	1888	3.7	5.3	11159	12.5	90.4
	1	4.30	23	2605	6.0	2.7	21871	15.8	89.7
	3	4.28	22	2484	6.0	3.8	20723	12.0	93.7
19	7	3.75	10	8720	8.7	4.2	20018	10.5	98.1
	0	5.40	47	526	2.0	3.8	14019	7.0	94.9
	1	5.30	42	605	2.8	3.4	20794	6.5	96.2
20	3	5.58	38	618	2.6	3.7	18517	2.6	94.4
	7	2.72	39	585	3.1	3.9	17129	4.8	98.3

NOTES: * Sampled in E/B lane only (All test sections)
 ** Sampled in W/B lane only (All test sections)

Table 6. FRICTION NUMBER VALUES (At 80km/h)

Test Sec- tion No.	L			A			N			E			Average of Both Lanes		
	Westbound						Eastbound								
	Years			After			Construction								
	0	2	6	0	2	6	0	2	6	0	2	6	0	2	6
1	37	36	38	36	NA	38	37	NA	38	37	NA	38	37	NA	38
2	35	30	33	34	NA	34	35	NA	34	35	NA	33	35	NA	33
3	36	34	38	35	NA	37	35	NA	37	35	NA	38	35	NA	38
4	33	33	38	30	NA	36	32	NA	36	32	NA	37	32	NA	37
5	31	28	32	30	NA	31	31	NA	31	31	NA	32	31	NA	32
6	33	37	41	35	NA	40	34	NA	40	34	NA	40	34	NA	40
7	30	26	29	32	NA	28	31	NA	28	31	NA	28	31	NA	28
8	29	26	29	30	NA	27	29	NA	27	29	NA	28	29	NA	28
9	30	26	32	31	NA	33	30	NA	33	30	NA	32	30	NA	32
10	33	29	34	35	NA	37	34	NA	37	34	NA	36	34	NA	36
11	37	36	41	37	NA	39	37	NA	39	37	NA	40	37	NA	40
12	35	37	39	35	NA	38	35	NA	38	35	NA	38	35	NA	38
13	33	32	37	33	NA	33	33	NA	33	33	NA	35	33	NA	35
14	28	25	30	30	NA	29	29	NA	29	29	NA	29	29	NA	29
15	29	24	27	27	NA	25	27	NA	25	28	NA	26	28	NA	26
16	38	34	39	37	NA	38	37	NA	38	37	NA	39	37	NA	39
17	34	33	40	33	NA	39	33	NA	39	17	NA	40	17	NA	40

Note: NA = Not Available

**Fig. 1 LAYOUT OF TEST SECTIONS
AND TYPES OF AGGREGATES USED IN TRIAL MIXES**
(Highway 7, Ontario)

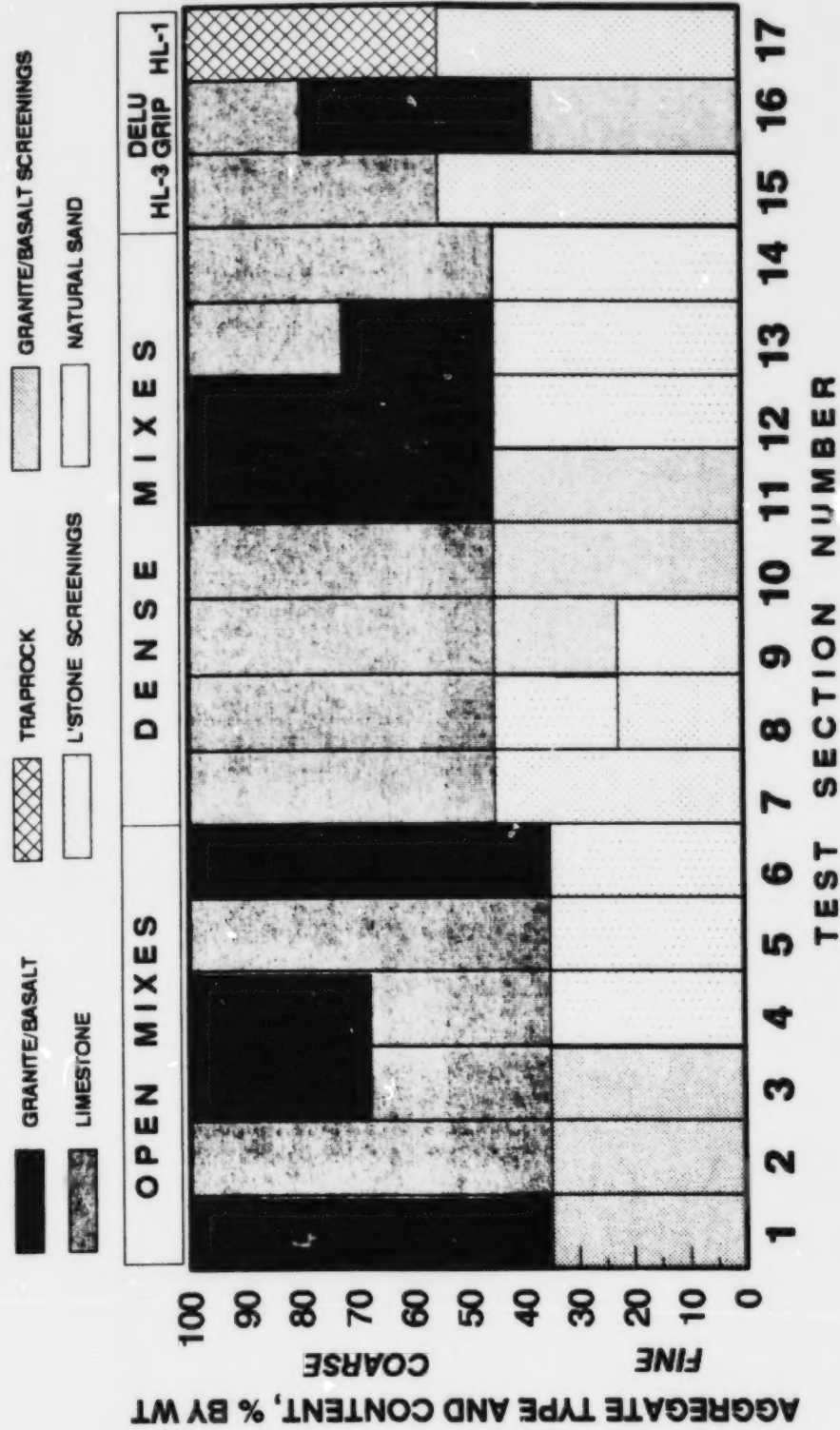


Fig. 2 AGGREGATE GRADATION CHART: MIXES 1 TO 6

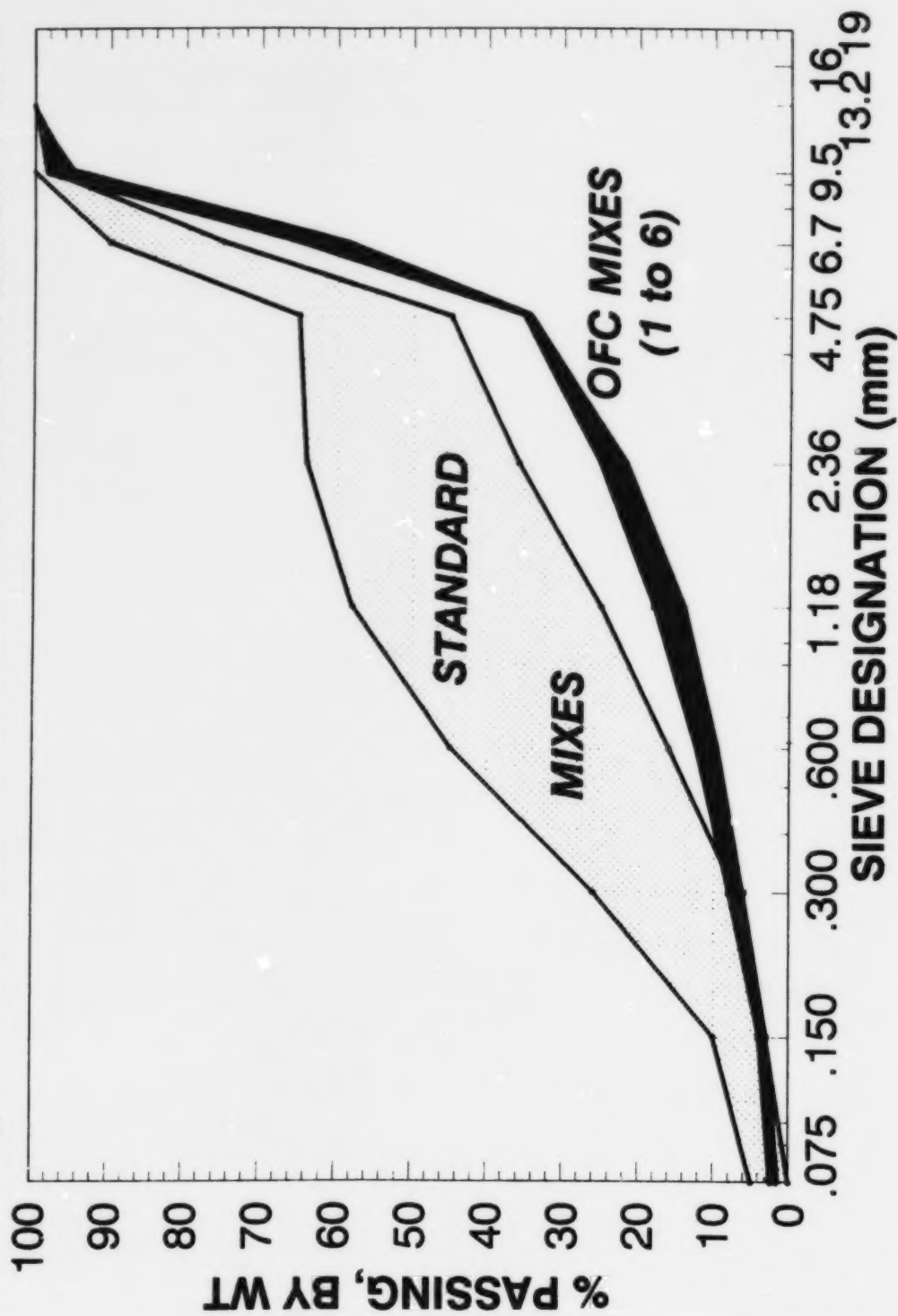


Fig. 3 AGGREGATE GRADATION CHART: MIXES 7 TO 12

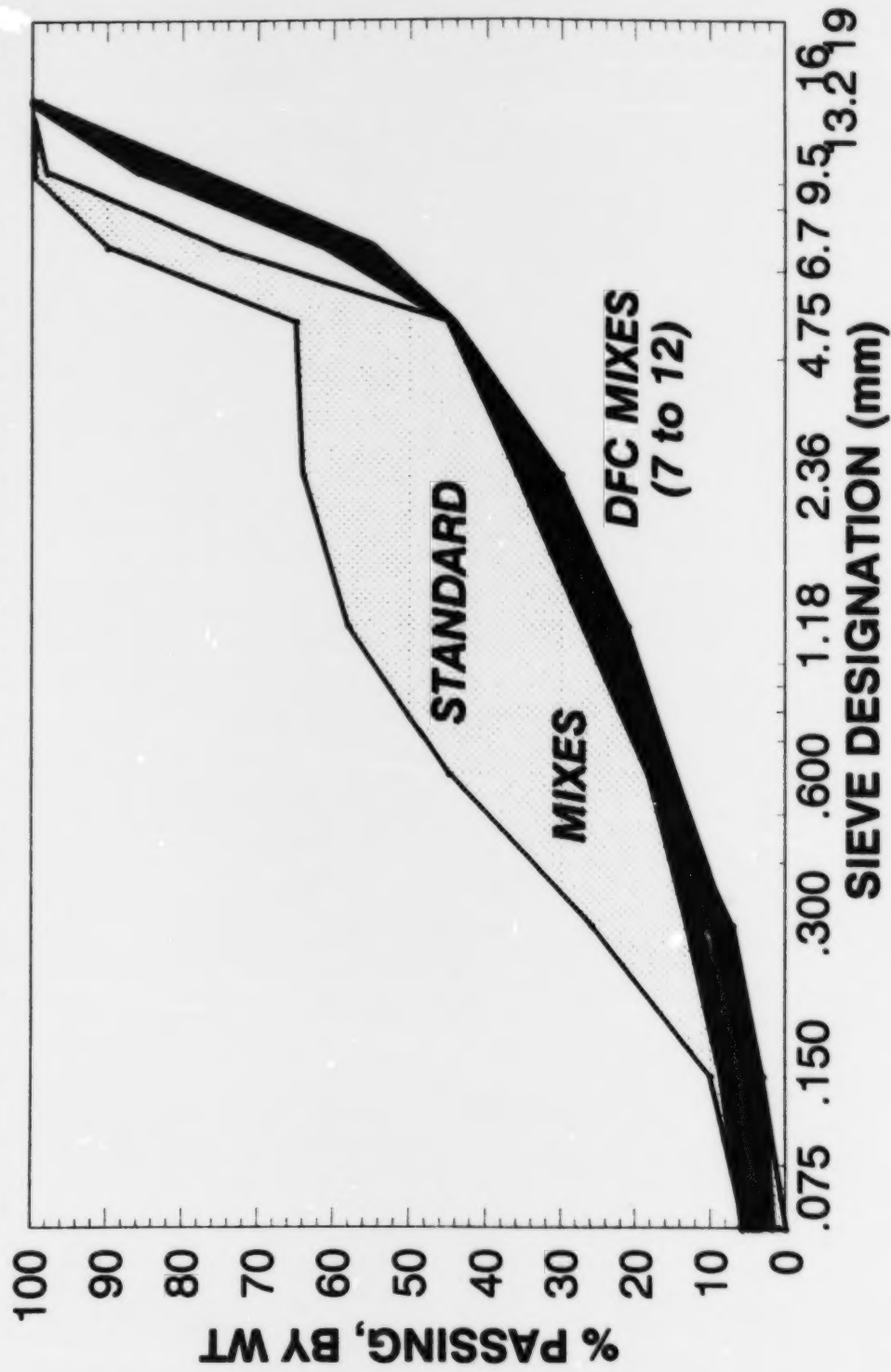


Fig. 4 AGGREGATE GRADATION CHART: MIXES 13 TO 17

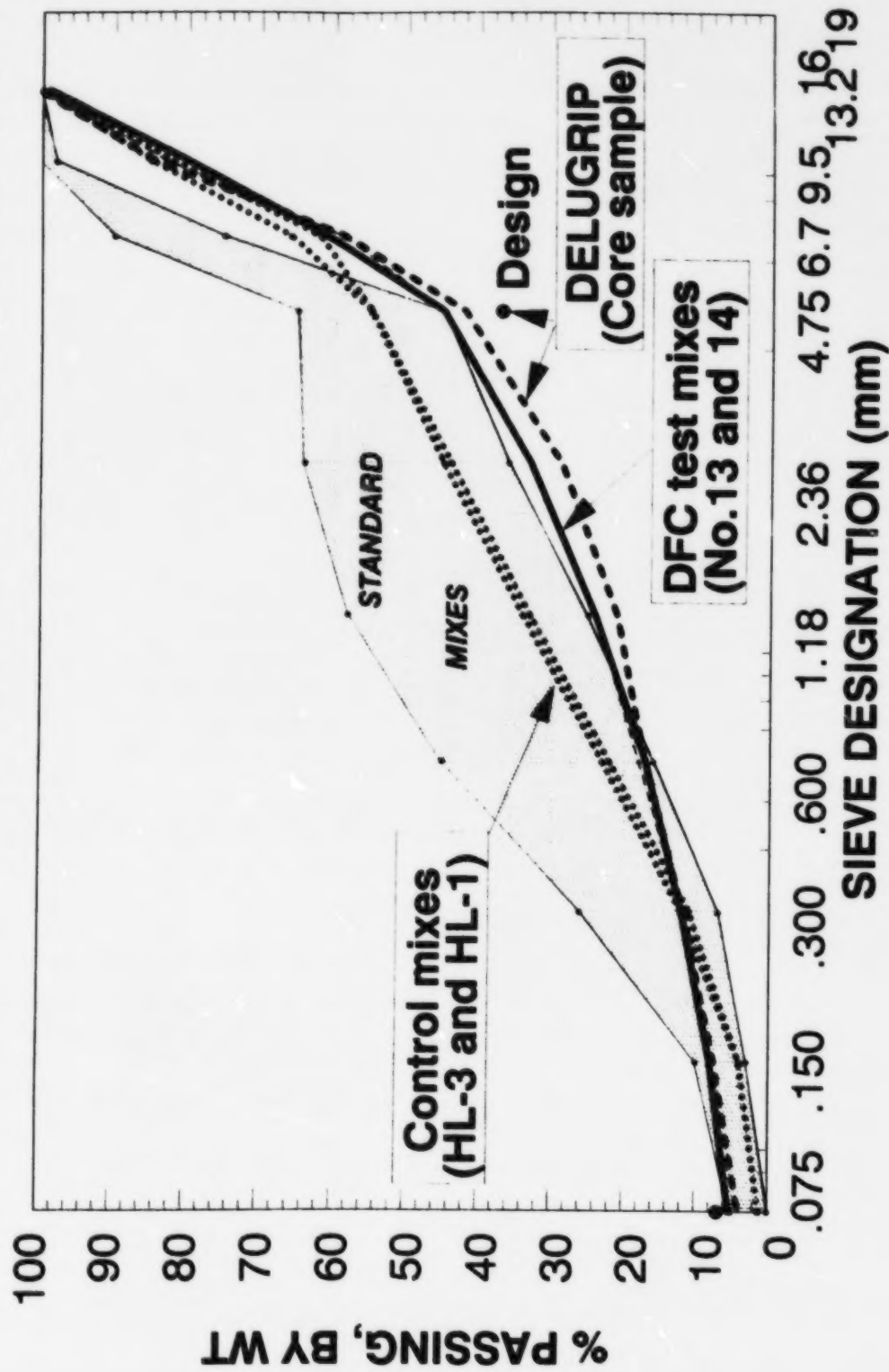


**Fig. 5 PENETRATION OF RECOVERED ASPHALT CEMENT
vs AIR VOIDS CONTENT IN PAVEMENT**

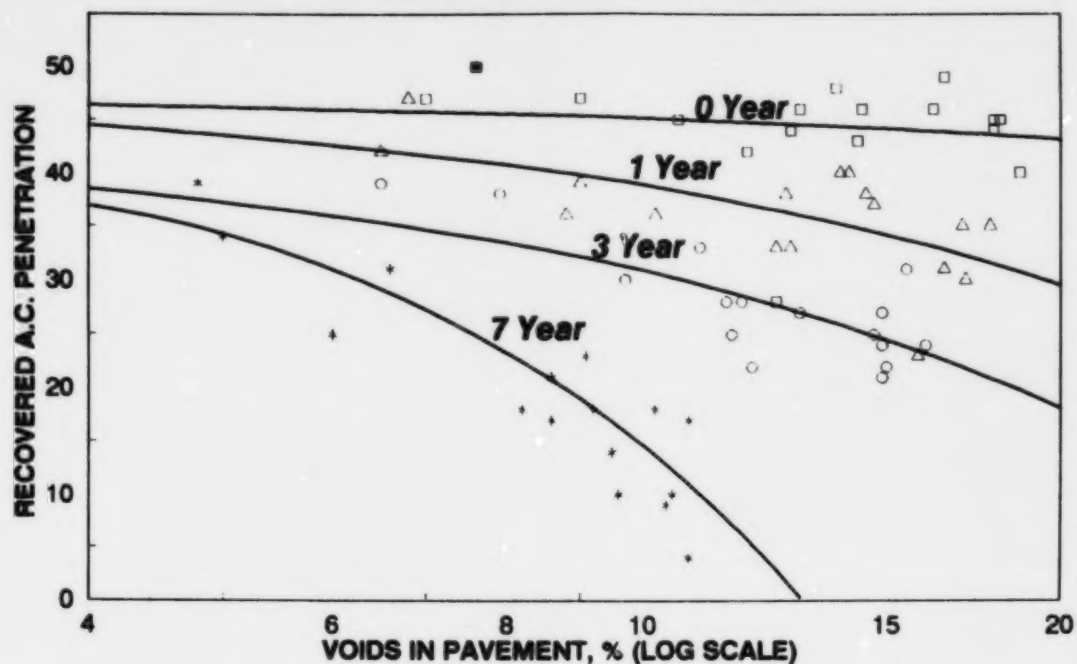


**Fig. 6 VISCOSITY OF RECOVERED ASPHALT CEMENT
vs AIR VOIDS CONTENT IN PAVEMENT**

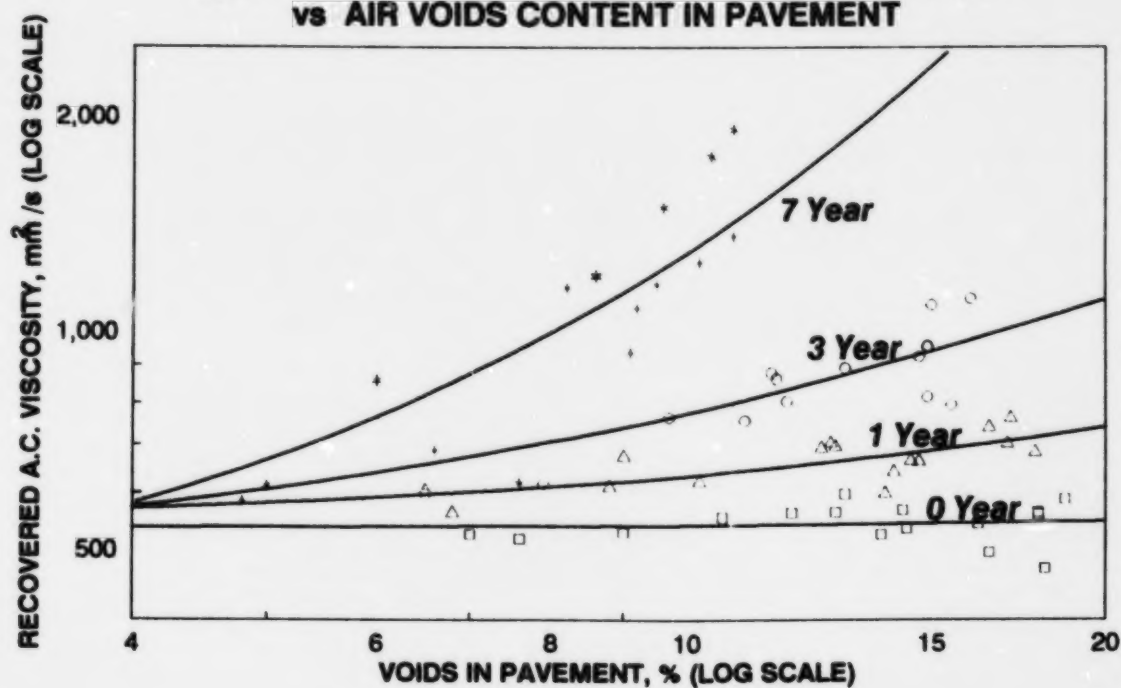


Fig. 7 MIX DESIGN STABILITY vs V.M.A.

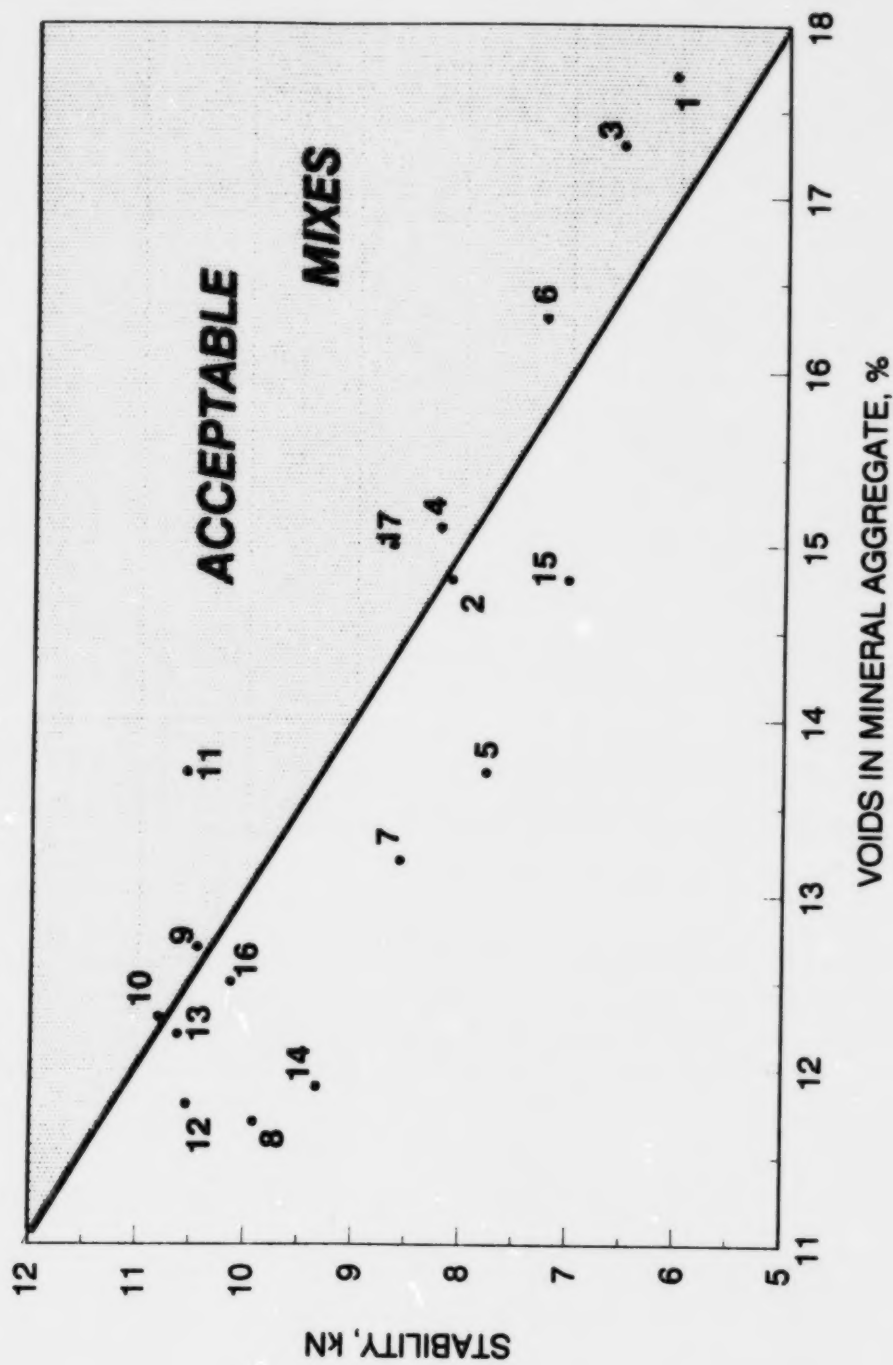


**Fig. 8 RELATION OF FRICTION NUMBER AT 80 km/h
TO PROPORTION OF HARD CRUSHED AGGREGATE IN MIX**

